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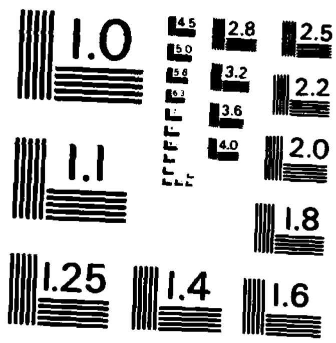
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US Army Corps of Engineers

AD-A160 695

REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-GT-2

# IMPROVEMENT OF LIQUEFIABLE FOUNDATION CONDITIONS BENEATH EXISTING STRUCTURES

by

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

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Final Report





REMR RESEARCH PROGRAM

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The following two letters used as part of the number designating technical reports of research published under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program identify the problem area under which the report was prepared

| | Problem Area | | Problem Area |
|---|---|---|---|
| CS | Concrete and Steel Structures | EM | Electrical and Mechanical |
| GT | Geotechnical | EI | Environmental Impacts |
| HY | Hydraulics | OM | Operations Management |
| CO | Coastal | | |

For example, Technical Report REMR-CS-1 is the first report published under the Concrete and Steel Structures problem area.



COVER PHOTOS

TOP Near failure of the old Lower San Fernando Dam, a Los Angeles Department of Water and Power project, as a result of liquefaction induced by the 1971 San Fernando earthquake.

BOTTOM – Settlement of a crane foundation in Japan as a result of earthquake-induced liquefaction, May 1983.

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# REPORT DOCUMENTATION PAGE

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

| | | |
|---|---|---|
| Admixture stabilization | Liquefaction | Soil reinforcement |
| Compression | Pore-water pressure relief | Thermal stabilization |
| In-situ deep compaction | Remedial treatments | |
| Injection and grouting | Soils | |

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The stabilization and improvement of liquefiable soils beneath existing structures is currently feasible within the state of knowledge and assumptions concerning liquefiable soils and earthquake excitation. This can be done directly or indirectly to mitigate the effects of liquefaction and to assure safe performance. At the present time there has been essentially no experience with remedial action in liquefiable soils at existing structures and no general method is applicable for all conditions. Each site is unique and will

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20. ABSTRACT (Continued).

require specific engineered solutions. This report presents and briefly discusses methodologies that have been deemed potentially applicable for remediating liquefiable soils beneath existing structures. A comprehensive bibliography is included on the feasible methods. The most important factors for construction in choosing remedial methods/techniques are: (a) the verifiability of improvement and stabilization, and (b) the assurance that the method itself will not create unsafe and unstable conditions under static and dynamic loading.

## PREFACE

This investigation was performed by the Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), US Army Engineer Waterways Experiment Station (WES), for the Headquarters, US Army Corps of Engineers (HQUSACE), during the period October 1982 to November 1984. The investigation was conducted under Civil Works Investigational Studies (CWIS) Work Unit 32219 and Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program Work Unit 32275, "Remedial Improvement for Liquefiable Foundations." Messrs. Richard F. Davidson and Arthur H. Walz, HQUSACE, were Technical Monitors for this work.

The REMR Overview Committee, which approved this study, consisted of Mr. John R. Mikel (DAEN-CWO-M), Mr. Bruce L. McCartney (DAEN-CWH-D), and Dr. Tony C. Liu (DAEN-ECE-D). Coordinator for the Directorate of Research and Development was Mr. Jesse A. Pfeiffer, Jr. (DAEN-RDC), and the REMR Program Manager was Mr. William F. McCleese, Concrete Technology Division, Structures Laboratory, WES.

This report was prepared by Mr. Richard H. Ledbetter, EEGD, under the supervision of Dr. Arley G. Franklin, Chief, EEGD, and Mr. Clifford L. McAnear, Chief, Soil Mechanics Division, GL, and under the general supervision of Dr. William F. Marcuson III, Chief, GL.

Commanders and Directors of WES during the preparation of this report were COL Tilford C. Creel, CE, and COL Robert C. Lee, CE; Technical Director was Mr. Fred R. Brown. During the publication of this report, COL Allen F. Grum, USA, was Director of WES; Dr. Robert W. Whalin was Technical Director.

CONTENTS

## CONVERSION FACTORS NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| kips (force) per square inch | 6,894.757 | kilopascals |
| tons (force) | 8,896.444 | newtons |

# IMPROVEMENT OF LIQUEFIABLE FOUNDATION CONDITIONS BENEATH EXISTING STRUCTURES

## PART I: INTRODUCTION

### Background

1. Some existing US Army Corps of Engineers earth dams, pumping plants, and locks on alluvial foundations in areas of moderate to high seismic hazard may require remedial action to improve the earthquake safety of their foundations. At present, studies are being conducted at specific Corps of Engineers dams to investigate the potential for liquefiable soil foundation materials. Many river levees are founded on potentially liquefiable material, and a stabilization method/technique is needed. The only options presently available are: (a) accepting the risk of catastrophic failure, (b) abandoning the project, and (c) replacing the structure. No experience with remedial actions at existing dams or structures founded on liquefiable foundation materials exists.

### Purpose

2. The purposes of the present study are to: (a) determine what relevant experiences exist in treating liquefiable soil conditions and may be applicable for use beneath existing structures, (b) determine the feasibility and effectiveness of techniques for improving liquefiable foundation conditions in order to assure safety against earthquake excitation, and (c) develop guidelines for laboratory and field experiments.

## PART II: POSSIBLE COURSES OF ACTION

3. The first course of action upon the identification of a potential foundation seismic stability problem beneath a structure is to verify that a problem exists. This is accomplished by evaluating: (a) the conservatism of the analytical techniques employed, (b) the conservatism of the predicted maximum credible earthquake, and (c) the adequacy of the soil strength determinations both in the field and laboratory. Once a seismic stability problem has been verified, the danger of failure, public safety, and courses of action to mitigate the threat must be considered. Table 1 summarizes possible courses of action for structures founded on liquefiable soil. These actions will either reduce the risk of failure or assure that the consequences of a damaging earthquake will be tolerable. These courses of action are discussed by Marcuson and Franklin (1983).

4. The last action listed in Table 1 is the subject of this report, i.e., improve the liquefiable soil foundation conditions directly or indirectly to assure the safe performance of structures founded on them in the event of earthquake excitation.

Table 1

Possible Courses of Action for Structures on Liquefiable Foundations

| Method | Comments |
|---|---|
| No action | Public reaction would be strong in the case of a major structure such as a dam because the public is not prepared to accept risk-based designs and judgments. The data base is weak on earthquake potentials for risk analysis |
| Regulate access to the structure and areas that would be affected by a failure | For a dam, the public reaction would be strong and it is not practical in an urban area |
| For a dam, restrict reservoir level<br>(a) Lower pool for a safe freeboard<br>(b) Permanently empty the reservoir | Safety is improved and the risk of complete failure is reduced. Public reaction would probably be strong because of regional, social, and economic impact. The advantages and uses of the dam for navigation, recreation, and power generation may be lost |
| Construct buttresses<br>(a) Earthen materials<br>(b) Retaining walls above and below ground<br>(1) Concrete<br>(2) Sheet pile<br>(3) Mixed in place with admixtures<br>(4) Double wall system | Buttresses can be designed and placed against structures to prevent movement and slope failure. The liquefaction potential of the foundation material can be reduced beneath the buttress prior to construction. The weight of an earthen buttress additionally increases the liquefaction resistance by increasing effective confining pressures in the foundation. Buttresses can be constructed upstream and downstream against a dam |
| For a dam, increase the height | Additional freeboard of a dam can be obtained; however, effective freeboard remaining after deformation and/or a flow slide would be difficult to reliably predict at present |

(Continued)

Table 1 (Concluded)

| Method | Comments |
|---|---|
| For a dam, construct a detention dam downstream | Cost would be less than a new structure and in the event of a dam failure it would only need to serve as a levee to retain water for a few days or weeks until a safe pool lowering could be carried out. A potential stability problem for a detention dam exists if a tsunami-like wall of water impacts on the dam. No experience exists in designing for such an impact and the structure should have a very high freeboard. Public reaction may be strong |
| For a dam, construct a replacement structure at either the same or a new location | A new structure can be designed and constructed to resist almost any earthquake shaking except in epicentral regions of large earthquakes of magnitude 8.0 and larger. This approach would (a) be very expensive, (b) cause economic impact if no reservoir usage, and (c) require a new environmental impact statement. Public reaction would probably be strong |
| Improvement of liquefiable soil foundation conditions directly or indirectly to assure the safe performance of structures founded on them in the event of earthquake excitation | These methods are summarized in Table 2. The methods are for direct in-situ improvement of liquefiable soils. However, combinations of these methods, including the methods in this table, can be used to indirectly improve the liquefiable conditions by mitigating, confining, and preventing detrimental consequences, thereby assuring safe performance of structures. Site conditions have been classified into three cases; Case 1 is for beneath structures and the indirect improvement methods are most applicable, Case 2 is for the not-under-water free field adjacent to a structure and the indirect improvement methods are most applicable, and Case 3 is for the under-water free field adjacent to a structure and the direct improvement methods are most applicable |

## PART III: METHODS FOR IMPROVING LIQUEFIABLE FOUNDATION CONDITIONS

5. Four basic approaches may be used to preclude damage caused by liquefaction: (a) increase the density of the soil, (b) increase the effective confining pressure, (c) provide protection to structures while liquefaction is allowed to occur, and (d) use particulate or chemical grouting to increase stiffness and fill the voids to prevent the orientation of soil particles to a denser state. Approaches a, b, and d improve the soil conditions to prevent liquefaction. Cyclic loading laboratory liquefaction tests show that if the effective confining pressure can be increased, the greater will be the number of cycles of dynamic shear stress required to induce liquefaction. This behavior is also true for increasing density.

6. Table 2* summarizes methods for improvement of liquefiable soil foundation conditions. The methods are for direct in-situ improvement. However, combinations of these methods, including the methods in Table 1, can be used to indirectly improve the liquefiable conditions and reduce damages by mitigating, confining, and preventing detrimental consequences, thereby assuring safe performance of structures.

7. In this study and in Table 2, site conditions have been classified into three cases: Case 1 is for beneath structures and indirect improvement methods are most applicable, Case 2 is for the not-under-water free field adjacent to a structure and the direct improvement methods are most applicable, and Case 3 is for the under-water free field adjacent to a structure and the direct improvement methods are most applicable. If water levels are lowered partially or completely for improvements at a structure such as a dam, all of the methods for Case 2 in Table 2 will be applicable to the upstream side of the dam. Also for the under-water free field, some methods are noted in Table 2 which are feasible with special high-cost techniques.

8. In assessing potential methods, a criterion was necessary for judging whether a method could sufficiently improve soil conditions to prevent or mitigate liquefaction occurrence. Technical Manual TM 5-818-1/AFM 88-3, Chapter 7 (Headquarters, Departments of the Army and the Air Force, 1983)

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* A table of factors for converting non-SI customary units of measurement used in Table 2 and elsewhere in this report to SI (metric) units is presented on page 3.

Table 2

Improvement of Liquefiable Soil Foundation Conditions

| Method | Principle | Most Suitable Soil Conditions/Types | Maximum Effective Treatment Depth | Economical Size of Treated Area | Ideal Properties of Treated Material* | Applications** | Case† | Relative Costs†† |
|---|---|---|---|---|---|---|---|---|
| In-Situ Deep Compaction | | | | | | | | |
| (1) Blasting | Shock waves and vibrations cause limited liquefaction, displacement, remolding and settlement to higher density | Saturated, clean sands, partly saturated sands and silts after flooding | >40 m Solymar (1984) | Any size | Can obtain relative densities to 70-80%; may get variable density, time-dependent strength gain | Induce liquefaction in controlled and limited stages and increase relative density to potentially nonliquefiable range | 2 3 | Low ($2.00-$4.00/m$^3$) |
| (2) Vibratory probe (a) Terraprobe (b) Vibro-rods (c) Vibro-wing | Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density | Saturated or dry clean sand; sand | 20 m routinely (ineffective above 3-4 m depth) >30 m sometimes Mitchell (1981) Vibro-wing-40 m Broms and Hansson (1984) | >1000 m$^2$ | Can obtain relative densities of 80% or more. Ineffective in some sands | Induce liquefaction in controlled and limited stages and increase relative density to potentially nonliquefiable range. Has been shown effective in preventing liquefaction | 2 3 | Moderate ($6.00-$13.00/m$^3$) |
| (3) Vibro-compaction (a) Vibroflot (b) Vibro-Compozer system (c) Soil Vibratory stabilizing method | Densification by vibration and compaction of backfill material of sand or gravel | Cohesionless soils with less than 20% fines | >30 m Solymar et al. (1984) | >1000 m$^2$ | Can obtain high relative densities (over 85%), good uniformity | Induce liquefaction in controlled and limited stages and increase relative densities to nonliquefiable condition. Is used extensively to prevent liquefaction. The dense column of backfill provides (a) vertical support, (b) drains to relieve pore water pressure and (c) shear resistance in horizontal and inclined directions. Used to stabilize *slopes and strengthen potential failure surfaces or slip circles* | 1 2 3‡ | Low to moderate ($6.00-$9.00/m$^3$) |
| (4) Compaction piles | Densification by displacement of pile volume and by vibration during driving, increase in lateral effective *earth pressure* | Loose sandy soils; partly saturated clayey soils; loess | >20 m Nataraja and Cook (1983) | >1000 m$^2$ | Can obtain high densities, good uniformity. Relative densities of more than 80% | Useful in soils with fines. Increases relative densities to nonliquefiable range. Is used to prevent liquefaction. Provides shear resistance in horizontal and inclined directions. Useful to stabilize slopes and strengthen potential failure surfaces or slip circles | 1 2 3 | Moderate to high |
| (5) Heavy tamping (dynamic compaction) | Repeated application of high-intensity impacts at surface | Cohesionless soils best, *other types can also be* improved | 30 m (*possibly deeper*) Ménard and Broise (1975) | >3300 m$^2$ | Can obtain high relative *densities, reasonable* uniformity. Relative densities of 80% or more | Suitable for some soils with *fines, usable above and be*low water. In cohesionless soils, induces liquefaction in controlled and limited stages and increases relative density to potentially nonliquefiable range. Is used to prevent liquefaction | 2 3 | Low ($0.60-$6.00/m$^3$) |

(Continued)

* SP, SW, or SM soils which have average relative density equal to or greater than 85 percent and the minimum relative density not less than 80 percent are in general not susceptible to liquefaction (TM 5-818-1). D'Appolonia (1970) stated that for soil within the zone of influence and confinement of the structure foundation, the relative density should not be less than 70 percent. Therefore, a criteria may be used that relative density increase into the *70-90 percent range is in general* considered to prevent liquefaction. These properties of treated materials and applications occur only under ideal conditions of soil, moisture, and method application. The methods and properties achieved are not applicable and will not occur in all soils.

** Applications and results of the improvement methods are dependent on: (a) soil profiles, types, and conditions, (b) site conditions, (c) earthquake loading, (d) structure type and condition, and (e) material and equipment availability. Combinations of the methods will most likely provide the best and most stable solution.

† Site conditions have been classified into three cases. Case 1 is for beneath structures, Case 2 is for the not-under-water free field adjacent to a structure, and Case 3 is for the under-water free field adjacent to a structure.

†† The costs will vary depending on: (a) site working conditions, location, and environment, (b) the location, area, depth, and volume of soil involved, (c) soil type and properties, (d) materials (sand, gravel, admixtures, etc.) equipment, and skills available, and (e) environmental impact factors. The costs are average values based on: (a) verbal communication from companies providing the service, (b) current literature, and (c) literature reported costs updated for inflation.

‡ 3 *means the method* has potential use for Case 3 with special techniques required which would increase the cost.

Table [illegible] (Continued)

| Method | Principle | Most Suitable Soil Conditions Types | Maximum Effective Treatment Depth | Economical Size of Treated Area | [illegible] Properties of Treated Material | Applications | Case | Relative Costs |
|---|---|---|---|---|---|---|---|---|
| (6) Displacement/compaction grout | Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure | All soils | Unlimited | Small | Grout bulbs within compressed soil matrix. Soil mass as a whole is strengthened | Increase in soil relative density and horizontal effective stress. Reduce liquefaction potential. Stabilize the ground against movement | 1<br>2<br>3 | Low to Moderate<br>([illegible]-[illegible]/m³) |
| | | | Compression | | | | | |
| (7) Surcharge/buttress | The weight of a surcharge/buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation | Can be placed on any soil surface | --- | >1000 m² | Increase strength and reduce compressibility | Increase the effective confining pressure in a liquefiable layer. Can be used in conjunction with vertical and horizontal drains to relieve pore water pressure. Reduce liquefaction potential. Useful to prevent movements of a structure and for slope stability | 2<br>3 | Moderate if vertical drains used |
| | | | Pore-Water Pressure Relief | | | | | |
| (8) Drains<br>(a) Gravel<br>(b) Sand<br>(c) Wick<br>(d) Wells (for permanent dewatering) | Relief of excess pore-water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains.) Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps | Sand, silt, clay | Gravel and sand >30 m<br>*Depth limited by vibratory equipment*<br>Wick >45 m<br>Morrison (1982) | >1500 m²<br>*Any size for wick* | Pore-water pressure relief will prevent *liquefaction* | Prevent liquefaction by gravel drains. Sand and gravel drains are installed vertically; however, wick drains can be installed at any angle. Dewatering will prevent liquefaction but not seismically induced settlements | Gravel and sand<br>2<br>Δ‡<br>Wick<br>1<br>2<br>3 | Sand and gravel [illegible]<br>([illegible]-[illegible])<br>Wick<br>([illegible]-[illegible])<br>Dewatering very expensive |
| | | | Injection and Grouting | | | | | |
| (9) Particulate grouting | Penetration grouting - fill soil pores with soil, cement, and/or clay | Medium to coarse sand and gravel | Unlimited | Small | Impervious, high strength with cement grout. Voids filled so they cannot collapse under cyclic loading | Eliminate liquefaction danger. Slope stabilization. Could potentially be used to confine an area of liquefiable soil so that liquefied soil could not flow out of the area | 1<br>2<br>3 | Lowest of grout methods<br>($[illegible]-$40.00/m³) |
| (10) Chemical grouting | Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate | Medium silts and coarser | Unlimited | Small | Impervious, low to high strength. Voids filled so they cannot collapse under cyclic loading | Eliminate liquefaction danger. Slope stabilization. Could potentially be used to confine an area of liquefiable soil so that liquefied soil could not flow out of the area. Good water shutoff | 1<br>2<br>3 | High<br>$[illegible]-$[illegible]/m³ |
| (11) Pressure-injected lime | Penetration grouting - fill soil pores with lime | Medium to coarse sand and gravel | Unlimited | Small | Impervious to some degree. No significant strength increase. Collapse of voids under cyclic loading reduced | Reduce liquefaction potential | 1<br>2<br>3 | Low<br>$10.00/m³ |

(Continued)

‡ Δ means the method has potential use for Case 3 with special techniques required which would increase the cost

(Sheet [illegible])

Table 2 (Continued)

| Method | Principle | Most Suitable Soil Conditions Types | Maximum Effective Treatment Depth | Economical Size of Treated Area | [illegible] Properties of Treated Material | Applications | [illegible] | [illegible] |
|---|---|---|---|---|---|---|---|---|
| (12) Electrokinetic injection | Stabilizing chemicals move [illegible] by electro-osmosis or colloids into pores by electrophoresis | Saturated sands, silts, silty clays | Unknown | Small | Increased strength, reduced compressibility, soils treated so they cannot [illegible] under [illegible] loading | [illegible] | [illegible] | Expensive |
| (13) Jet grouting | High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels | Sands, silts, clays | Unknown | Small | Solidified columns and walls | Slope stabilization by providing shear resistance in horizontal and inclined directions which strengthens potential failure surfaces or slip circles. A wall could be used to confine an area of liquefiable soil so that liquefied soil could not flow out of the area | [illegible] | High $[illegible]/m³ |
| **Admixture Stabilization** | | | | | | | | |
| (14) Mix-in-place piles and walls | Lime, cement, or asphalt introduced through rotating *auger* or special in-place mixer | Sand, silts, clays, all soft *or loose inorganic* soils | >20 m (60 m obtained in Japan) Mitchell (1981) | Small | Solidified soil piles or walls of relatively high strength | Slope stabilization by providing shear resistance in horizontal and inclined directions which strengthens potential failure surfaces or slip circles. A wall could be used to confine an area of liquefiable soil so that liquefied soil could not flow out of the area | 1<br>2<br>3 | High $250.[illegible]–$6[illegible]/m³ |
| **Thermal Stabilization** | | | | | | | | |
| (15) In-situ vitrification | Melts soil *in place* to create an obsidianlike vitreous material | All soils and rock | >30 m Verbal from Battelle Laboratories | Unknown | Solidified soil piles or walls of high strength Impervious, more durable than granite or marble, compressive strength, 9-11 ksi, splitting tensile strength, 1-2 ksi | Slope stabilization by providing shear resistance in horizontal and inclined directions which strengthens potential failure surfaces or slip circles. A wall could be used to confine an area of liquefiable soil so that liquefied soil could *not* flow out of the area | 1<br>2<br>3 | Moderate $53.00–$70.00/m³ |
| **Soil Reinforcement** | | | | | | | | |
| (16) Vibro-replacement stone and sand columns (a) Grouted (b) Not grouted | Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by *vibro* techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout | Sands, silts, clays | >30 m **Limited by vibratory equipment** | >1500 m² **Fine-grained soils** >1000 m² | Increased vertical and horizontal load carrying capacity. Density increase in cohesionless soils. Shorter drainage paths | Provides, (a) vertical support, (b) drains to relieve pore water pressure, and (c) shear resistance in horizontal and inclined directions. Used to stabilize slopes and strengthen potential failure surfaces or slip circles | 1<br>2<br>3‡ | Moderate $11.00–$70.00/m³ |

(Continued)

‡ 3 means the method has potential use for Case 3 with special techniques required which would increase the cost.

(Sheet 3 of 4)

Table 2 (Concluded)

| Method | Principle | Most Suitable Soil Conditions/Types | Maximum Effective Treatment Depth | Economical Size of Treated Area | Ideal Properties of Treated Material | Applications | Case | Relative Costs |
|---|---|---|---|---|---|---|---|---|
| | | | | | | For grouted columns, no drainage provided but increased shear resistance. In cohesionless soil, density increase reduces liquefaction potential | | |
| [illegible] Root piles, soil nailing | Small-diameter inclusions used to carry tension, shear, compression | All soils | Unknown | Unknown | Reinforced zone of soil behaves as a coherent mass | Slope stability by providing shear resistance in horizontal and inclined directions to strengthen potential failure surfaces or slip circles. Both vertical and angled placement of the piles and nails | 1<br>2<br>3 | Moderate to high |

(Sheet 4 of 4)

has a criterion for excluding the need for detailed liquefaction analysis. This criterion states that SP, SW, or SM soils (Unified Soil Classification System) which have average relative densities equal to or greater than 85 percent and a minimum relative density not less than 80 percent are in general not susceptible to liquefaction. D'Appalonia (1970) states that for soil within a zone of influence and confinement of a structure foundation, the relative density should not be less than 70 percent. Therefore, a criterion established for this study is that a method producing a relative density increase into the 70 to 90 percent range is generally considered capable of preventing liquefaction.

9. Figure 1 presents the applicable grain-size ranges for the liquefiable soil improvement methods. Also superimposed on the figure is the grain-size range most sensitive to liquefaction (Bhandari, 1981). The grain-size ranges for the improvement methods are from TM-5-818-1/AFM 88-3, Chapter 7 (Headquarters, Departments of the Army and the Air Force, 1983) and American Society of Civil Engineers (ASCE) (1978b).

10. Each structure and structure site is unique and will probably require specific engineered solutions for the particular liquefaction circumstances and conditions. No general method or approach is believed applicable for all conditions and structures, and combinations of the methods will most likely provide the best and most stable solution. The selection of potential methods for a site improvement as well as the applications and results of the methods will depend on:

a. The location, area, depth, and volume of soil involved.

b. The soil type(s), properties, and conditions.

c. The site conditions.

d. Earthquake loading.

e. Structure type and condition.

f. Economic and social effects of the structure.

g. Availability of necessary materials such as sand, gravel, and admixtures.

h. Availability of equipment and skills.

11. The selection of potential methods also depends on the cost of the method/technique and the length of time needed to stabilize a site. The costs for site improvements will vary depending on the above factors (a through h) as well as:

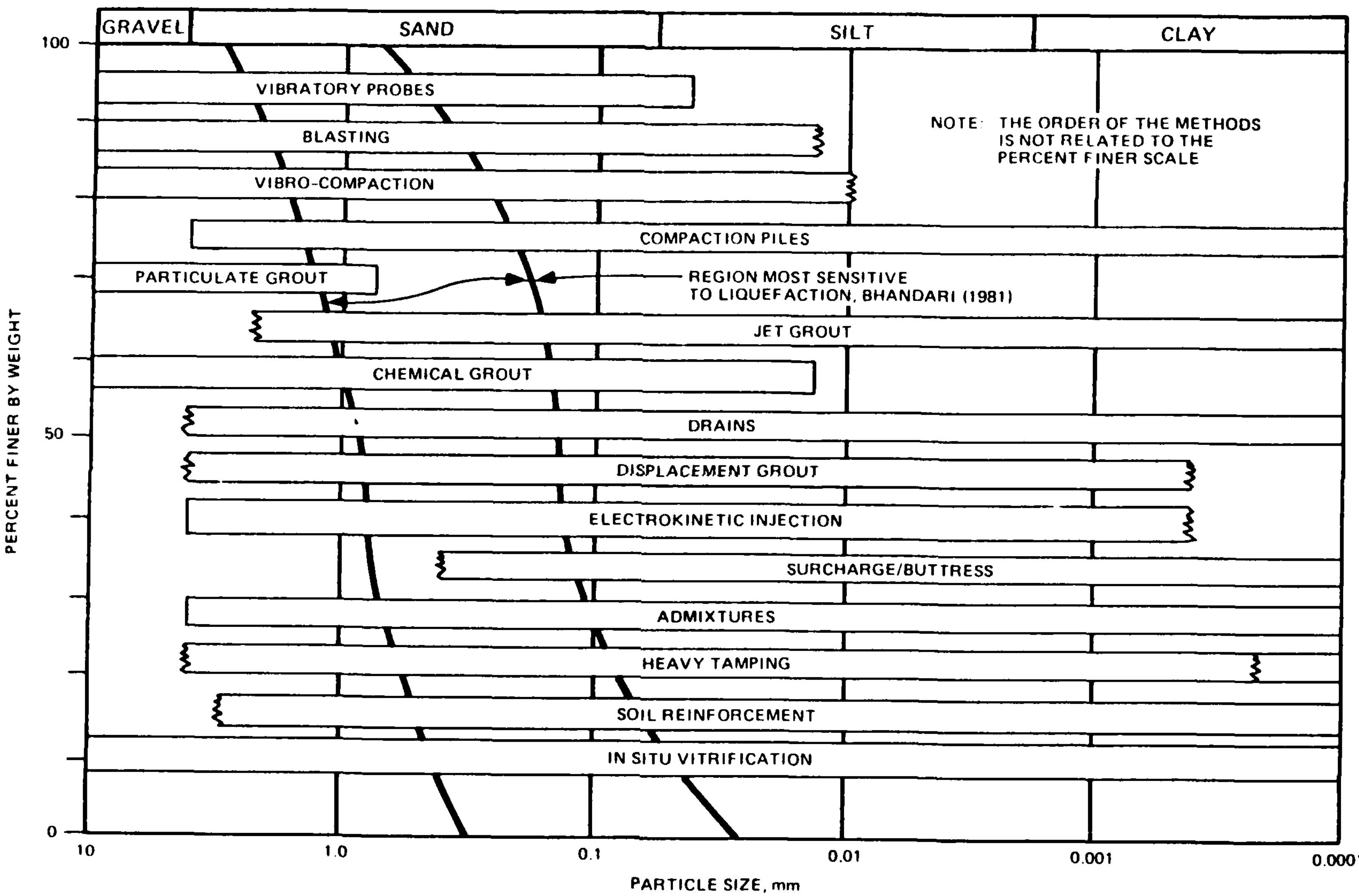


Figure 1. Applicable grain-size ranges for liquefiable soil improvement methods

a. Site working conditions, location, and environment.

b. Environmental impact factors.

12. At the present state of the art, field tests must be conducted to insure that a selected improvement method is applicable under specific site and soil conditions, to verify that the method will perform its intended functions, and to show that the method will not threaten the safety and stability of the structure. Additionally, several improvement methods may be investigated at a site field test to determine the most applicable and economical method(s).

13. Listed in Table 2 are the most suitable soil conditions/types for which the methods are applicable. For example, vibro-compaction methods may not be successful in sands with more than 20 percent fines (Saito, 1977). It must also be emphasized that to increase the relative density of a soil from 50 to 75 percent is much easier than increasing it from 75 to 85 percent. Harder, Hammond, and Ross (1982) report on an unsuccessful test program where vibroflotation was used in an attempt to densify a liquefiable silty sand foundation of an existing embankment. Some of the sand samples contained more than 20 percent fines; however, more than half of the samples were in the gradation range best suited to densification by vibroflotation. Effectiveness of the vibroflotation method was measured by the standard penetration test and cone penetration test. Test results showed that the soil was not densified and was even made looser at some places. Possible explanations for the failure were the high silt content of the sand and/or an overlying silt/clay cap.

14. The most important factors for consideration in choosing an improvement method is the verifiability of improvement and stabilization and, that the method will not cause safety problems. Not only must the function/behavior of a method be verified at a field test location, but the final improvement product and results for the structure foundation must be verified in order to assure safe performance. Field verification techniques are addressed in Part IV of this report.

15. In applying remedial methods to dams, the complex interrelationships must be considered within a dam concerning its core, shells, transition zones, filter zones, drains, and impermeable blankets as well as the interactions of a dam with its foundation, appurtenant structures, and reservoir margin. Extreme caution must be exercised to avoid creating a new defect

in the process of applying remedial treatment methods to dams. Treatment methods and operations must be carefully specified and monitored to prevent damage to dams. After remedial treatments, the stability and safety of a dam must be assured under static and water loads. Table 3 presents precautions, for each remedial method of Table 2, that must be kept in mind when planning, designing, and executing treatment methods for a dam.

## Combinations of Methods

16. Combinations of improvement methods in Tables 1 and 2 may provide the best solution to a stability problem. Any combination will have to be engineered to solve a specific problem, and the results of individual methods as well as the combination will have to be verified for a structure's stability.

17. As a hypothetical example of potential method combinations, consider an existing earth dam underlain by a liquefiable layer of loose sand which extends 30 m upstream and 30 m downstream. The layer is 3 m thick and at a depth of 3 m. Liquefaction analysis has shown the material to be liquefiable if subjected to the maximum credible earthquake. Stability analysis of the dam shows slope failures will occur both upstream and downstream due to the liquefaction in the foundation. The following paragraphs present potential combinations of the improvement methods from Table 2 for possible remedial treatment of the dam problems. No significance is attached to the order in which the combinations are presented.

### Potential treatment combination 1

18. Grout (method 6, 9, 10, or 12) the liquefiable sand beneath the dam to prevent liquefaction and sufficiently densify the sand upstream and downstream of the toes to prevent liquefaction for a distance which safely includes the induced stress spread with depth. A method for densifying the sand downstream could be chosen from methods 1, 2, 3, 4, 5, and 6. If the reservoir pool is lowered for remedial treatment, the downstream densification methods would also be applicable for the upstream area. If the reservoir pool is not lowered, method 3 would not be applicable. Densification methods 1 through 5 produce controlled limited liquefaction and have an effect from 1 to 4 m radially out from the point of application

Table 3

Potential Impacts of Remedial Methods on Dam Safety

Under Static and Water Loads Only

| Method | Precautions |
|---|---|
| **In-Situ Deep Compaction** | |
| 1. Blasting applicable to Cases 2 and 3* | When used near and beneath toe areas, potential hazards include induced sliding, slope failures, and damage to drains from motions and differential settlements |
| 2. Vibratory probe applicable to Cases 2 and 3 | When used near and beneath toe areas, potential hazards include: disturbance of and creation of new drainage paths; slides, slope failures, and damage to drains from differential settlements |
| 3. Vibro-compaction applicable to Cases 1, 2, and 3 | See method 2. For Case 1, damage can be caused to impermeable blankets, transition zones, filter zones, and drains. Holes can have rapid drawdown conditions and cause instability. These hazards can lead to piping and hydraulic fracturing |
| 4. Compaction piles applicable to Cases 1, 2, and 3 | See methods 2 and 3 |
| 5. Heavy tamping (dynamic compaction) applicable to Cases 2 and 3 | See method 1 |
| 6. Displacement/compaction grout applicable to Cases 1, 2, and 3 | For Case 1, holes can have the problems of methods 2 and 3; heavy differential movements, and fractures can cause damage to impermeable blankets, transition zones, filter zones, and drains. Drilling fluids can cause hydraulic fracturing. These hazards can lead to to piping and hydraulic fracturing |
| **Compression** | |
| 7. Surcharge/buttress applicable to Cases 2 and 3 | Differential settlements can damage impermeable blankets, transition zones, filter zones, and drains with results of piping and hydraulic fracturing |

(Continued)

* Site conditions have been classified into three cases; Case 1 is for beneath structures, Case 2 is for the not-under-water free field adjacent to a structure, and Case 3 is for the under-water free field adjacent to a structure.

Table 3 (Concluded)

| Method | Precaution |
|---|---|
| Pore Water Pressure Relief | |
| 8. Drains applicable to Cases 1, 2, and 3 | See methods 2 and 3 |
| 9. Particulate grouting applicable to Cases 1, 2, and 3 | See method 6 |
| 10. Chemical grouting applicable to Cases 1, 2, and 3 | See method 6 |
| 11. Pressure-injected lime applicable to Cases 1, 2, and 3 | See method 6 |
| 12. Electrokinetic injection applicable to Cases 1, 2, and 3 | Holes can have the problems of methods 2 and 3 |
| 13. Jet grouting applicable to Cases 1, 2, and 3 | Holes and potential settlements can have the problems of methods 2 and 3 |
| Admixture Stabilization | |
| 14. Mix-in-place piles and move walls applicable to Cases 1, 2, and 3 | Holes, trenches, and differential ments can have the problems of methods 2 and 3. Continuous mix-in-place walls near the toes can potentially cause slope instability |
| Thermal Stabilization | |
| 15. In-Situ Vitrification applicable to cases 1, 2, and 3 | Settlements due to decreased volume of voids could cause damage to impermeable blankets, transition zones, filter zones, zones, and drains which could lead to piping and hydraulic fracturing |
| Soil Reinforcement | |
| 16. Vibro-replacement stone and sand columns applicable to Cases 1, 2, 3 | See methods 2 and 3 |
| 17. Root piles, soil nailings applicable to Cases 1, 2, and 3 | See methods 2 and 3 |

of the method; therefore, they can be used fairly close to and may possibly include the toe area of the dam.

Potential treatment combination 2

19. Densify the liquefiable sand upstream and downstream of the dam toes using the same methods as in combination 1. Construct underground walls through the liquefiable layer upstream and downstream in the toe areas. The walls are for containing the sand and preventing it from flowing out from under the dam in the event of liquefaction. Walls are constructed by methods 14 and 15. In this case, potential settlement following liquefaction has to be considered including the dam's toleration of the settlement. Confinement of the sand could also be achieved by grouting.

20. Protection of structures with underground walls (sheet piles) for containing the liquefied soil was proven effective during the 1964 Niigata earthquake in Japan (Yoshimi, 1980). Underground walls have been modeled and studied by Yoshimi and Tokimatsu (1977) with remarkably beneficial results in preventing damage to a structure.

21. For treatment combination 2, the dam slopes also would be stabilized. The downstream slope stabilization would be by a method or methods chosen from 3, 4, 6, 7, 8, 13, 14, 15, 16, or 17. If the reservoir pool is not lowered, methods 3, 8, and 16 would not be applicable for stabilizing the upstream slope. If method 16 does not use a grouted column, drainage is an added benefit of this method. Design methods and field tests for the seismic stability of slopes using stone columns are contained in Englehardt and Golding (1975) and Mitchell (1981). These design methods are applicable to other types of columns and walls used for slope seismic stability. A design for gravel drains for preventing liquefaction is given by Seed and Booker (1977). Slope stability methods using piles, including root piles, are given in Mitchell (1981), Murray (1980), and Dash and Jovino (1980). The stabilizing effects of underground walls and buttressses can be determined from standard slope stability analysis techniques.

Potential treatment combination 3

22. Drainage of the liquefiable sand layer to relieve excessive pore water pressure during an earthquake and improve stability of the potential slope shear zone may be all that is necessary to protect the structure. Drains from method 8 can be used to stabilize the free field upstream and downstream areas and also beneath portions of the dam slopes. Gravel and/or

sand drains additionally provide horizontal and inclined shear resistance for slope stability. If the drains silt or clog, the buildup of pore-water pressure will blow out the silt and clear the drain.* Stabilization for upstream and downstream slope stability can be chosen from the same methods presented above in combination 2. Design methods were also referenced in combination 2.

Potential treatment combination 4

23. This combination is to grout or densify the liquefiable sand partially beneath the dam and sufficiently upstream and downstream of the toe for a distance which safely includes the induced stress spread with depth. Also, the upstream and downstream slopes are stabilized. The methods listed in the previous combinations are applicable for this one.

## Methodologies

24. The following paragraphs present brief discussions of the methods listed in Table 2. References that are particularly relevant are used in the discussions. More complete descriptions of techniques and equipment can be found in the Bibliography, which is divided into specific method headings.

In-situ deep compaction

25. The deep compaction methods are blasting, vibratory probes, vibrocompaction, compaction piles, heavy tamping, and displacement/compaction grout. In-situ densification with these methods can be used to minimize the possibility of and prevent liquefaction under seismic loading. Factors that affect the use of these methods to achieve the necessary improvements include:

a. Soil type, gradation, and saturation.

b. Location of water table.

c. Initial relative density.

d. Characteristics of the method used.

26. The deep compaction methods are capable of obtaining relative densities in the 70 to 85 percent range with densification being achieved by movement of the soil particles into a denser packing. Localized liquefaction is induced for an effective distance of 1 to 4 m from the energy source

* From H. B. Seed's discussion at WES in Spring of 1983.

(Mitchell, 1981), which allows these methods to be used fairly close to a structure. The vibratory probes and compaction methods are best suited for soils with less than 20 to 25 percent fines (Headquarters, Departments of the Army and Air Force, 1983); whereas blasting, compaction piles, and heavy tamping can be used in more silty and clayey soils.

27. Blasting. Densification by blasting is the detonation of buried explosives in saturated soils. Partly saturated soils may need to be flooded. Little densification is likely to result above a 1-m depth and loosened material may remain around blast points. Empirical design procedures are in Headquarters, Departments of the Army and Air Force (1983) and Mitchell (1981).

28. Vibratory probes. Vibratory probes of the Terraprobe and Vibro-rods types are cylindrically shaped probes which are driven into the ground by a vibratory pile driver and which compact by vibration during withdrawal. Terraprobe was developed in the United States (Anderson, 1974) and Vibro-rods was developed in Japan by Saito (1977). The Vibro-Wing method uses a long steel rod with wings attached and a heavy vibratory hammer mounted to the top. Insertion to a desired depth is by the vibratory hammer and then vertical vibrations are transmitted to the soil by the vibrator. The time required is reported to be significantly less than required for other deep compaction methods (Broms and Hansson (1984)). This is a new device developed in Sweden (Massersch and Lindberg, 1984). These probes are best suited for cohesionless clean sands. Vibratory probe densification is a popular method in Japan for mitigating damage from liquefaction (Saito, 1977, and Yoshimi, 1980). The methods are also used elsewhere in the world to treat liquefiable soils (Kirsch and Chambosse, 1981; Dobson and Slocombe, 1982). Design methodology for the vibratory probe use can be found in Thorburn (1975).

29. Vibro-compaction. Vibro-compaction methods include the Vibroflot, the Vibro-Composer System, and the Soil Stabilizing Method. These methods involve vibratory displacement of the soil radially outward from the center of a depth vibrator as it penetrates under its own weight and the power of vibration. The cylindrical hole is then backfilled in stages with sand or gravel which is compacted with the depth vibrator at each stage, thereby forming a compacted pile of sand or gravel. The Vibroflot is a depth probe producing horizontal vibrations, while the Soil Vibratory Stabilization Method combines both the vertical vibration of a vibratory driving hammer

and the horizontal vibrations of a depth probe vibrator. In the Vibro-Compozer method, a vibrator at the top of a casing pipe drives it to a desired depth and sand backfill in stages is forced down and out of the casing by compressed air upon repeated withdrawal and reinsertion until the casing is out of the ground.

30. Vibro-compaction methods are very popular for treating liquefiable soils (Yoshimi, 1980; Saito, 1977; Engelhardt and Golding, 1975; Harder, Hammond, and Ross, 1982; and Dobson and Slocombe, 1982). Design methodology can be found in Mitchell (1981), Headquarters, Departments of the Army and Air Force (1983), and Thorburn (1975).

31. Compaction piles. Compaction piles involve the densification of soils and increase in lateral effective earth pressure by displacement of the pile volume and by vibrations during the pile-driving operation. Sands, silts, and clays can be densified by driving piles. Relative densities sufficient to prevent liquefaction have been achieved (Iyengar, 1981; Nataraja and Cook, 1983). A design method for pile diameter, spacing, and volume reduction of the soil is described by Iyengar (1981).

32. Heavy tamping. Heavy tamping (Ménard and Broise, 1975) is repeated impacts of a very heavy weight dropped from heights up to 40 m. Weights up to 200 tons have been used (Mitchell, 1981). Heavy tamping can also be carried out under water with a special fluted pounder (ASCE, 1978c), where a crane mounted on a barge is used and a water depth of 8 to 10 m is required for a sufficient drop height. Surface craters are formed by heavy tamping and they have to be filled and compacted. Heavy tamping has been used to reduce the liquefaction potential of loose sands and silts (ASCE, 1978c; Bhandari, 1981). Relative densities of 70 to 90 percent can be obtained which are the amounts necessary to resist seismic induced liquefaction. Design methodology for the mass, height of fall, and depth of compaction are given in Bhandari (1981), ASCE (1978c), and Ménard and Broise (1975).

33. Displacement/compaction grout. Displacement/compact on grouting compacts the soil radially as a bulb of low-slump grout is injected under high pressure. The stiff nature of the grout causes it to remain in a mass and to densify and compact the soil surrounding it. Additionally, the horizontal effective stress is increased by the process. Displacement grout bulbs increase the soil's relative density and increase the shear resistance

in horizontal and inclined directions similar to the benefits of vibro-compacted displacement columns. Displacement grout bulbs can potentially be placed from any inclined direction (may be limited by depth of overburden) to reach soils not accessible from the vertical direction, such as beneath a structure.

Compression

34. A surcharge or buttress can be used to increase the liquefaction resistance of a material and to react against a structure to prevent movements and for stabilizing potential slope failure. The weight of the surcharge/buttress increases the effective confining pressures in the foundation and adds more mass to be moved by potential earthquake-induced deformations and slope instabilities. Design of surcharge/buttress size and weight can be made with conventional geotechnical static and dynamic analysis techniques for embankments and slope stabilities.

Pore-water pressure relief

35. Gravel or rock drain systems can be used to prevent liquefaction by dissipating pore-water pressures almost as fast as they are generated in earthquake-induced cyclic loading. Analytical and experimental studies have shown gravel drains to be very effective (Sasaki and Tanigughi, 1982; Seed and Booker, 1977; and Yoshimi, 1980). Design methodologies are also presented in these references. A liquefiable soil layer can also be kept permanently dewatered with pumps.

36. Sand and wick drains may be used to supplement gravel drains and to relieve existing excess pore-water pressure in a confined layer of soil susceptible to liquefaction. Wick drains (Morrison, 1982) have comparable permeabilities to sand drains and have the advantage that they may be installed at any angle.

Injection and grouting

37. Grouting and injection can be used to stabilize a liquefiable soil and prevent liquefaction by filling the soil voids. This increases the strength and stiffness of the soil and primarily prevents the orientation of soil particles into a denser state, thereby preventing the increase in pore-water pressure and liquefaction inducement from seismic cyclic loading (Headquarters, Departments of the Army and Air Force, 1983; D'Appolonia, 1970; and Yoshimi, 1980).

38. Particulate and chemical grout. There are two classes of grout--particulate and chemical. Particulate grouts are usually made with portland cement. Lime slurry for pressure injection is also a particulate grout. Because of the silt-size particles in these grouts, soils finer than medium to coarse sand cannot have their pores injected with particulate grouts. The soil pores or voids have to be larger than the grout particles for permeation.

39. In order to penetrate the voids of fine-grained soils, chemical grouts must be used. The major factor controlling groutability is the viscosity of the chemical-water solution. The most common classes of chemical grouts are silicates, resins, lignins, and acrylamides (Headquarters, Departments of the Army and Air Force, 1983). Effective ranges of soil particle sizes for chemical groutability are shown in Figure 2 (Hayward Baker, 1982) with the most sensitive liquefaction region (Bhandari, 1981).

40. Dupas and Pecker (1979) conducted an extensive laboratory testing program (about 1,000 samples involved) to assess the characteristics of behavior during static loading, dynamic loading, and specifically the liquefaction potential of cement-treated sand. They found that only a small amount (5 percent) of cement is required to prevent liquefaction. It was also

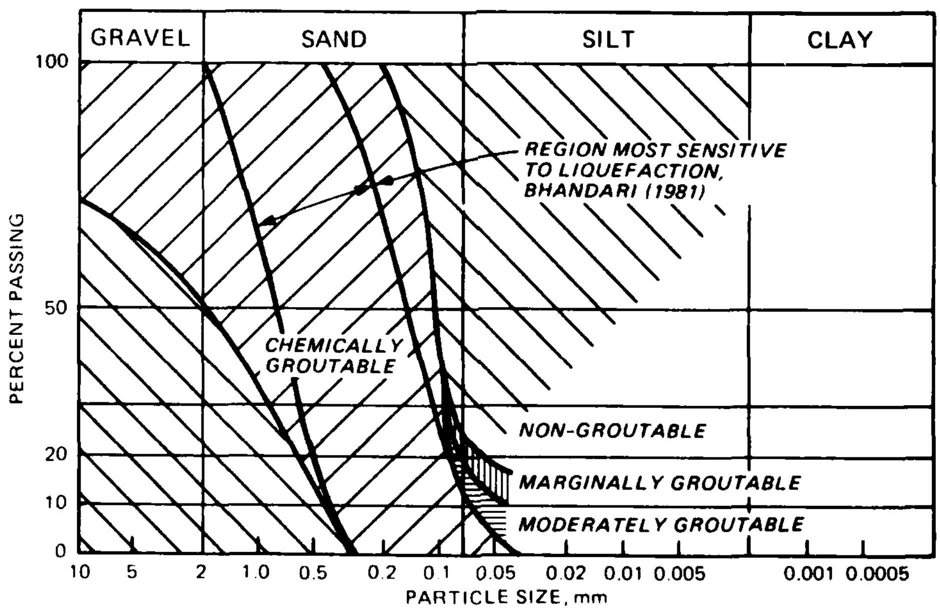


Figure 2. Effective ranges of soil particle sizes

shown that no chemical bonds are destroyed during cyclic loading, and that the samples are able to withstand very severe dynamic loads.

41. Sawada et al. (1981) conducted field observations and measurement tests on cement-mixed sand test sites. The test sites experienced 17 earthquakes ranging in magnitude from 3.5 to 7.2. They found that the maximum amplitudes of the horizontal motions were reduced at the treated test sites to about one-fourth of those at an untreated control site. This indicates a remarkable reduction effect of cement-treated soil on horizontal earthquake motions. The soil improvement made a slight reduction of the vertical motions.

42. Perez, Davidson, and Lacroix (1982) reported on a program involving particulate and chemical grouting tests and final chemical grouting for Mississippi River Lock and Dam 26. The soil was an alluvial sand and eight grout types were compared. Field and laboratory tests on undisturbed grouted sand samples were conducted. It was found feasible to inject silicate grouts with good penetration and coverage of the volume of soil necessary to be treated. The grouting operation was achieved without causing significant vertical or horizontal soil displacement or excessive pore-water pressure. The required strength and stiffness of the foundation soil was achieved.

43. Rosenfarb and Hackman (1981) state that chemical grouts have been successfully applied to soils which are subject to dynamic and cyclic loading conditions, such as machine foundations, railroad track structures, and subway tunnels. They also report on a laboratory study of static and dynamic (cyclic) test behavior of a silicate grout-stabilized sand. Results were an increase of dynamic strengths and moduli with increasing silicate content and in all cases the dynamic strengths were lower than the static strengths. Parish, Baker, and Rubright (1983) reported good penetration and areal coverage in sand and gravel with chemical grout improvement for Pittsburgh's new subway.

44. Grouting techniques. Commonly grouts are injected into the ground under pressure using either open-pipe or sleeve-pipe methods. Another technique is electrokinetic injection, which consists of dispersing the chemical grout through the soil by an electrical gradient (Mitchell, 1981). A unit electrical gradient can be more effective than a unit hydraulic gradient for moving fluids through fine-grained soils. Electrokinetic injection can be used where ordinary grouting techniques cannot be used (Mitchell,

1981); such as (a) in silty soils, (b) where there is lack of confinement preventing high grouting pressure, and (c) beneath structures.

45. Yamanouchi and Matsuda (1975) tested electrokinetic injection to fill the voids of a loose sand with a gel (silicate) and with colloidal material (bentonite and aluminum hydroxide). Test results showed a marked increase in resistance to liquefaction after treatment. Oncescu and Bally (1977) reported on strengthening the loess foundation beneath a theater using electrokinetic injection of a silicate.

46. Other types of grouting. In addition to particulate and chemical grouting, there are also displacement (discussed in paragraph 33) and jet grouting. For jet grouting high-speed jets are used at depth to excavate, inject, and mix the grout stabilizer with soil to form columns or panels. Columns up to 3 m in diameter are possible (Mitchell, 1981). Grouted columns or panels increase the shear resistance of the soil mass in horizontal and inclined directions similar to the benefits of other columnar resulting methods.

## Admixture stabilization

47. Admixture stabilization includes mixed-in-place soil and stabilizers to form columns and walls. Columns as large as 3 m in diameter are possible as above with jet grouting. This stabilization or improvement method consists of the mechanical mixing through rotating augers or special in-place mixers. Normal stabilizers mixed with soil are lime, cement, or asphalt (Mitchell, 1981). Most inorganic soils can be treated. The resulting product is solidified soil piles, columns, or walls of relatively high strength. These piles and columns provide shear resistance in horizontal and inclined directions. Design and analysis methods for vibro-compaction columns can be applied to these admixture columns. In-situ deep mixing methods have been developed in Sweden (Broms and Boman, 1979) and Japan (Endo, 1976).

## Thermal stabilization

48. The thermal stabilization method considered applicable to this study is in-situ vitrification of soil and/or rock (Timmerman and Lokken, 1983; Oma, Farnsworth, and Rusin, 1982). In this procedure soil and rock are melted in place to create an obsidian-like vitreous material. The method is applicable to all soils and rock and to depths greater than 30 m. Solidified-soil piles, columns, and walls of any size can be constructed. The vitreous material has very high strengths, 9 to 11 ksi compressive and

1 to 2 ksi splitting tensile strength. The vitreous material is impervious and more durable than granite or marble. Shear resistance in horizontal and inclined directions is provided by the vitreous material. Design analysis methods for other columnar techniques are applicable, such as referenced in the vibro-compaction method.

Soil reinforcement

49. Vibro-replacement stone and sand columns. These are columns constructed of compacted backfill gravel or sand in holes that may be formed by such means as jetting in fine-grained soil or by vibro techniques in cohesionless soils. Vibro-replacement columns can be placed in any soil with usual diameters of 0.6 to 1.0 m (Mitchell, 1981). The columns can be grouted to make them stronger or not grouted. These columns provide (a) vertical support, (b) drains to relieve pore-water pressure (if not grouted), and (c) shear resistance in horizontal and inclined directions. Stone columns have been used to prevent seismic-induced slope instability (Engelhardt and Golding, 1975). Stone columns can be designed and analyzed by the methodologies in Barksdale and Bachas (1983), Mitchell (1981), and others referenced under the vibro-compaction method and the pore-water pressure relief method.

50. Root piles and soil nailing. These are small-diameter inclusions in soil used to carry tension, shear, and compression. They can be used in most soils. Root piles are small-diameter piles in the range of 0.075 to 0.25 m (Mitchell, 1981), and are cast-in-place concrete with a reinforcing bar in the center. Soil nailing is accomplished by a series of reinforcing bars grouted into the ground.

51. Root piles and soil nailing reinforce a zone of soil that consequently behaves as a coherent mass which may be analyzed following usual geotechnical procedures (Mitchell, 1981). This reinforced soil mass has increased shear resistance and may act as a retaining wall for unreinforced soil. Applications and analysis of root piles for slope stability are in Murray (1980) and Dash and Jovino (1980). Design and analysis procedures for the columnar methods referenced in this part of the report may also be applicable to the behavior of a reinforced soil mass.

## PART IV: VERIFICATION OF IMPROVEMENTS

52. The determination of effectiveness is essential for remedial treatments for the improvement of liquefiable soil foundation conditions. Verifiability of the improvement is an important consideration in selecting treatment methods. At the present state of the art, field tests of a method must be conducted in order to verify the applicability and effectiveness of the in-situ improvement. Verification can be obtained by field measurements, laboratory tests, and analytical analysis. These three verification approaches must be used together to evaluate improvements.

### Field Measurement Techniques

53. Techniques for measuring the results and effective improvements of stabilization methods are (a) geophysical techniques and (b) penetration, in-situ strength tests, and monitoring. Complete descriptions can be found for the geophysical techniques in EM 1110-2-1802 (Office, Chief of Engineers, US Army, 1979) and for the penetration and strength tests in EM 1110-2-1907 (Office, Chief of Engineers, US Army, 1972), TM 5-818-1 (Headquarters, Department of the Army, 1983), ASTM Part 19, and references cited in the following paragraphs. Other references for use of the field measurement techniques in evaluating improvement methods are listed in the Bibliography under evaluation and verification.

#### Geophysical techniques

54. Determination of in-situ mass areal characteristics, as opposed to discrete point characteristics from laboratory testing and field penetration testing, is an important advantage of geophysical techniques. Techniques that are applicable to the evaluation of the improvement methods are:

- a. Seismic:
  - (1) Crosshole shear-wave velocity.
  - (2) Surface vibratory Rayleigh-wave velocity.
  - (3) Surface refraction shear-wave velocity.
  - (4) Surface refraction compression-wave velocity.
- b. Surface resistivity.
- c. Radar.

d. Nuclear moisture and density.

e. Downhole geophysical logging.

f. Microgravity.

g. Magnetics.

Techniques a through e have been used to evaluate stabilized soils. Microgravity and magnetic methods have high potential for surface mapping of a modified soil mass extent. Microgravity can determine density variations on the order of 0.04 $g/cm^3$ for a 3-m thickness and could be used to map the uniformity of a stabilized soil mass and locate potential low treatment zones. Surface resistivity, radar, and constant spaced surface refraction can also be used for mapping the extent of a modified soil mass.

55. Erchul and Gularte (1982) have studied and developed a method for evaluating the liquefaction potential of sand by electrical resistivity techniques. Perez, Davidson, and Lacroix (1982) found crosshole shear-wave velocities to be an effective means for evaluating grouting effectiveness. They concluded that the velocities were influenced not only by the type of grout but also, and more significantly, by the extent of grout penetration and that the measurements reflected the bulk properties of the mass through which the waves propagated. Parish, Baker, and Rubright (1983) used shear-wave velocity measurements to evaluate chemical grouting. Ground-probing radar has been used by Hayward Baker (1981) in evaluating grouting.

Penetration, in-situ strength tests, and monitoring

56. Field penetration and in-situ strength tests can be used to evaluate stabilized soil at discrete points or locations. These tests include:

a. Standard penetration test (SPT).

b. Cone penetration test (CPT).

c. Pressure meter test (PMT).

d. Vane shear.

e. Borehole permeability.

f. Piezometers.

g. Deformation gages.

h. Undisturbed sampling.

Most of these field tests techniques have been used and found applicable to evaluate the effectiveness of the soil stabilization techniques of Table 2, with the exception of the thermal stabilization method. Discussion of

the results from specific field test techniques that have been used can be found in the previously cited references of the soil improvement methods. The most widely used field tests are SPT, CPT, PMT, and piezometers. Perez, Davidson, and Lacroix (1982) used six of the above techniques on one project to evaluate grout effectiveness. The six tests were a, b, c, e, f, and g. Mitchell (1981 and 1984) discusses SPT and factors influencing it.

## Laboratory Tests

57. Laboratory evaluation of soil parameters and strengths should be conducted on undisturbed samples obtained both before and after a soil improvement method is used. Dynamic strength, damping, and liquefaction resistance changes are very important in assessing the improvement of a soil to resist seismic loading and liquefaction. Cyclic load tests of grouted sands have been referenced under grouting methods. Perez, Davidson, and Lacroix (1982) conducted laboratory investigations of grouted soil in addition to the above-referenced field tests.

## Analytical Analysis

58. The indices and strengths from field measurements and laboratory tests on improved soil should be used in new analyses to reassess the seismic stability of a structure and site due to the improvements. Analysis procedures have been previously referenced for the columnar, pile, and wall improvement methods. These include Mitchell (1981), Barksdale and Bachas (1983), Engelhardt and Golding (1975), Thorburn (1975), Murray (1980), Dash and Jovino (1980), D'Appalonia (1970), and Iyengar (1981). Density and strength improvements can be directly taken into account in conventional static and dynamic analysis techniques.

59. In some cases it may be desirable to analyze the effects of soil improvement techniques on the seismic stability and safe performance of a site and structure with dynamic effective stress analyses. The preferred analysis is one that takes into account changes in permeability, pore-water pressure behavior, volume change characteristics, dynamic shear modulus behavior, and damping behavior of the soil as these are affected by the improvement techniques. The analysis should also consider the possibly

changed non-linear responses of the modified soils. Ledbetter (1983) presents several models which could be used in dynamic effective stress analyses. The Finn model was recommended as currently the preferred dynamic effective stress method.

# PART V: LABORATORY AND FIELD EXPERIMENTAL STUDIES AND RECOMMENDATIONS

60. Although the applications and results of improvement methods are specifically soil and site dependent, laboratory and field general investigations and field demonstrations can be conducted to better define method limitations, improve the applicability and results, and improve verification technology. Topics for investigation are presented in the following paragraphs.

## Laboratory Investigations

61. The cyclic load dynamic response and characteristics of grout and admixture stabilized soils need to be investigated. Questions to be addressed and answered include:

- a. How does the cyclic loading liquefaction resistance change?
- b. How does the cyclic/dynamic shear strength change?
- c. How does the dynamic shear modulus change?
- d. How does the damping response change?
- e. Does the stabilizing agent degrade with cyclic loading and/or with time and environment?
- f. What are the minimum amounts of stabilizing agents required to significantly reduce the potential for or prevent liquefaction?
- g. Does the stabilizing agent cause a post-earthquake stability problem by adversely affecting the pore-water pressure behavior?

62. The investigation should be conducted with the different grouts and chemical stabilizing agents in current use. Preferably, a representative range of liquefiable soils should be used in the tests, but a general investigation could be conducted with only one liquefiable soil. In addition to the liquefiable soils, the investigation should include the effects of stabilizing agents in typical sands, gravels, and crushed stone used for sand and stone columns. This applies to the dynamic behavior of stabilized/grouted sand and stone columns. Cyclic triaxial and simple shear, centrifuge, and shake table laboratory testing could be used for this investigation.

63. Another topic for laboratory investigation is the very economical wick drain. Considerations should include the following:

a. Would wick drains be effective in relieving pore-water pressure to prevent liquefaction?

b. What optimum permeability is desired for wick drains in preventing liquefaction?

c. What soil gradation is best suited for use of wick drains?

d. Will fines clog/stop wick drains preventing relief of pore-water pressure, and what is the maximum percentage of fines allowable?

e. How do wick drains behave during earthquake cyclic shear loading and will they be torn apart?

f. It it does not presently exist, can the optimum wick drain be manufactured for relief of seismically induced pore-water pressures to prevent liquefaction?

This study could be conducted by laboratory testing with cyclic triaxial, centrifuge, or shake table equipment. Different soil gradations in the liquefiable range and different wick types should be used.

## Model Testing

64. Prototype behavior can be investigated under seismic loading by the use of models tested in a centrifuge and/or on a shake table. In the centrifuge test, the pore fluid viscosity should be scaled in order to obtain prototype pressure, time, and permeability behavior of the pore fluid (Eyton, 1982; Dean and Schofield, 1983; Dean and Lee, 1974; and Finn, Siddharthan, and Ledbetter, 1985). An embankment or structure incorporating a liquefiable layer in its foundation can be modeled with various improvement methods included. Tests could include the following:

a. A model embankment or dam with a wall for confining the liquefiable soil and with the slopes stabilized. As discussed in Part III under "Combinations of Methods," several methods can be tested for (1) a confining wall, and (2) stabilizing the slopes.

b. A grouted liquefiable layer beneath an embankment or structure.

c. Stone columns (grouted and ungrouted), piles, displacement grout bulbs, and formed-in-place piles and walls to provide shear resistance and prevent movements in either or both a liquefiable layer and a slope.

65. Another modeling topic is that if a vibro-compactor or vibrator could be adequately modeled or scaled down, it is feasible that the applicability and effectiveness of the vibratory methods could be determined in

laboratory testing for a given liquefiable soil. This would result in significant economic savings over field testing to determine the vibratory applicability and effectiveness at a given site.

66. An additional topic for both conventional soils laboratory and model studies is the investigation of the phenomenon and importance of behavior of induced changes in horizontal effective stresses caused by the deep compaction methods. Mitchell and Solymar (1984), Lukas (1980), and others report decreases in penetration resistance followed by time-dependent increases after the use of deep compaction methods. What are the mechanisms involved, and are the stress changes significant and permanent? Is the phenomena possibly a process of deconsolidation due to very high lateral forces from a compactor and then a re-consolidation or re-equilibration and stress adjustment with time? Laboratory studies could involve stress path testing to model deep compaction methods with the increase of lateral stresses greater than the vertical stress. The stress-strain and pore-water pressure behavior during and after a test could be monitored. Strength changes and stress-strain adjustments could be studies with time and after a loading test.

## Improvement Verification Investigations

67. A field test area needs to be established at a liquefiable soil location. The test area would be established for investigating and refining techniques for measuring the results and effective improvements of various treatment methods. Techniques including in-situ probing and boring and geophysical testing could be investigated for such factors as sensitivity, calibration, correlation, repeatability, accuracy, and applicability.

## Order of Priority of Recommended Studies

68. The following is the order of priority of needed investigative studies for remediating liquefiable soils:

a. Investigation of the significance of horizontal effective stress behavior with deep compactive methods.

b. Laboratory test techniques to replace field tests for determining the applicability of remedial methods for a given site, resulting in significant economic savings.

c. Wick drain applicability to liquefiable soils.

d. Dynamic response characteristics of various stabilizing agents for grouting and injection into a soil.

e. Improvement verification investigations.

## PART VI: CONCLUSIONS

69. The conclusions reached in this study are:

a. It is currently feasible within the state of knowledge and assumptions concerning liquefiable soils and earthquake excitation to stabilize and improve liquefiable soil foundation conditions directly or indirectly and mitigate effects of liquefaction in order to assure the safe performance of existing structures founded on them in the event of earthquake excitation.

b. There has been essentially no experience with remedial actions at existing structures founded on liquefiable foundation materials.

c. The most important factors for consideration in choosing remedial methods/techniques are the verifiability of improvement and stabilization and the assurance that the method itself will not create unsafe and unstable conditions under static and dynamic loading.

d. No general method is applicable for all conditions and structures. Each site is unique and will require specific engineered solutions for the particular circumstances and conditions.

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